Precast Tile Beam Floor

By Henry Giese and Charles T. Bridgman

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Agricultural Engineering Section

 Ames, Iowa
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SUMMARY AND CONCLUSIONS

Correct usage and economical placement of materials are important considerations in low-cost housing.

If the floor system described herein contributes toward these ends, it is because the materials are located where they may function to advantage and are placed there at a saving of materials and labor ordinarily used in making, placing and removing forms.

The design of the floor lends itself readily to a wide range of loading conditions. By using only one shape of beam tile it can be adapted to the requirements of light residences or heavy, warehouse loads. The five variables which influence the strength are:

1. Length of span.
2. Spacing of beams (as fixed by the length of span tile).
3. Depth of beam (as fixed by the depth of span tile).
4. Depth of beam (as fixed by the thickness of concrete topping).
5. Diameter of reinforcing steel.

This bulletin includes two series of tests, the first made with a beam tile as first conceived, and the second with a tile revised to eliminate the shortcomings of the first. In addition to the beam tests, several tests were made of slabs and completed floors to secure approval for construction purposes in several cities.

From these tests and from extensive construction experience, design formulas and suggestions for procedure have been developed and are included.

1. With one exception, all failures of the typical floor sections were caused by the yielding of the steel.
2. The compression area of the section was stronger than the tension even though only $\frac{1}{2}$ inch of concrete topping was used.
3. There were no shear, compression or bond failures in the testing except in the section using the 1" round rods.
4. Seven-eighths inch is the maximum diameter of the reinforcing rod which the beam tile used in these tests will accommodate.
5. Span tile, 24 inches long, can be used in all cases except for heavy loading conditions or spans longer than 19 feet.
6. Span tile, 4 inches deep, is satisfactory for most loading conditions.
7. The cost per square foot of the reinforced tile floor increases as the length of span increases.
8. The deflection at the design load, of spans under 20 feet, is much less than the allowable of $\frac{1}{360}$ of the span.
9. The usual formulas for the design of reinforced concrete T beams can be used for this type of reinforced tile floor.
Precast Tile Beam Floor

By Henry Giese and Charles T. Bridgman

The floor system described in this publication is the result of an attempt to obtain a floor of masonry materials, at a minimum cost in placing, which will successfully meet the loading requirements placed upon it.

Most systems of masonry floor require considerable unproductive labor and materials which add to the cost but which do not contribute to the effectiveness of the ultimate structure. Forms must be built and dismantled. Even, where moveable and reusable forms are employed, the cost of assembling, disassembling and moving add to the charges which must be ultimately placed against the cost of the floor. This overhead becomes proportionally larger if the forms do not find continuous use.

DEVELOPMENT OF REINFORCED MASONRY

The first known reinforced brick masonry was introduced in 1836 by Sir Marc Isambac Brunel (37) at Nine Elms, England. He constructed a brick beam 21 feet 4 inches long and reinforced it with iron hoops. For 2 years this beam supported approximately 24,000 pounds. In 1838, the beam was tested to destruction, failing at 68,328 pounds due to elongation of the steel.

So remarkable was this test that India saw the possibilities of reinforced brick masonry as a substitute for its expensive building materials. Extensive research was carried on by the government, and today India uses reinforced brick masonry more than any other country.

After spending several years in India, Mason Vaugh (36) applied some of his experiences to tests on reinforced brick masonry at the University of Missouri in 1928 and found that combination tile and brick beams and hollow brick beams carried loads “not greatly different” from the solid beams.

In 1932, John W. Whittemore and Paul S. Dear (37) of Virginia Polytechnic Institute investigated the performance
characteristics of reinforced brick masonry slabs. They concluded that such slabs perform in a manner similar to reinforced concrete slabs and are, therefore, theoretically and experimentally practical.

The Common Brick Manufacturers' Association of America has carried on extensive research on reinforced brick masonry in this country with the aid of such prominent engineers as Prof. R. H. Danforth of the Case School of Applied Science, Hugo Filippi (12) of Chicago, Judson Vodges (22) of Philadelphia and Major L. B. Lent of Cleveland and has recommended that the standard specifications for concrete and reinforced concrete be adopted to govern the design and construction of reinforced brick masonry.

Another clay product assumed importance in the design of combination tile and concrete floors in 1930. Then, D. E. Parsons and A. H. Stang (28) of the National Bureau of Standards made tests on composite beams and slabs of hollow tile and concrete. They concluded that the tile web was of more value in resisting deformation and deflection than an equal volume of concrete and that the compressive stresses in the shells of the hard tiles in contact with the concrete
ribs were greater than the compressive stresses in the adjacent concrete.

The Iowa Agricultural Experiment Station has long been active in endeavoring to use masonry materials where their high compressive strength could be used to advantage and to supplement the use of steel to carry the tensile stresses. Examples of this are shown in the Iowa silo (11), the masonry water supply tank (18) and the all masonry barn (16). In all instances effort has been made to get the material in place at the lowest possible cost. By eliminating the unproductive labor and the use of unproductive materials, it is believed that a floor can be obtained at satisfactory cost which will meet the requirements placed upon it.

IDEALS FOR FLOOR CONSTRUCTION

A floor construction, to obtain wide usage in low-cost housing projects, small commercial and industrial buildings, farm houses and farm service buildings, should possess several qualities.

First, the construction must be readily adaptable to a variety of design conditions without extensive engineering
Fig. 3. Tile beam floor used in a dairy barn.

Fig. 4. A masonry floor was used in the M. D. Judd residence, Mason City, Iowa.
service. The units of the floor must be so simple that the average builder will be able to lay out the floor plan.

Second, the floor design must be sufficiently flexible to meet a variety of loading conditions. This must be done without complicating the design data and with a minimum number of special units.

Third, the floor itself should be easily constructed and should require little special equipment for installation. The laying of the floor should be so simple that the average worker in the rural areas with little previous experience will be able to install it. Forming is expensive and requires skill to build so should be avoided or reduced to a minimum.

Fourth, the materials should be easily manufactured and distributed with few special or complicated units. The units should be of a size that can be handled easily.

THE T BEAM FLOOR

In the floor system shown in fig. 5, the load is carried largely by precast beams extending from wall to wall. The space between beams is taken up with span or filler tile. The entire surface is covered with concrete after the beams and span tile are in place. Tensile stresses in the beams are carried by steel bars imbedded in concrete “troughs” formed by the tile. The span tile and the concrete topping, which is considerably thicker directly over the beam, carry the compressive stresses.
Five factors affect the ability of the floor to carry superimposed loads:
1. Length of span.
2. Spacing of beams (as fixed by the length of span tile).
3. Depth of beam (as fixed by the depth of the span tile).
4. Depth of beam (as fixed by the thickness of concrete topping).
5. Diameter of reinforcing steel.

Thus, it may be seen that by using only one special beam tile shape and perhaps two lengths to accommodate various span lengths many load capacities are possible. It then also appears to meet our requirements for strength and simplicity. As the beams are cast, and later laid in place and the span tile laid on ledges formed on the precast beam, no form work is required on short spans. One line of stiffening shoring is used on medium length spans and two for long spans.

Two series of tests are herein described. The first deals briefly with beams made from the original tile. The second, and more complete series, gives the results obtained from the tile redesigned as a result of the first series of tests.

Additional information gained from field experiences provides design data and shows construction methods found to be effective and economical.

BEAM TEST—FIRST SERIES (5)

The shape of the beam tile was designed to provide the following requirements:
1. A form for the mortar necessary to bond the reinforcing steel to the tile.
2. Sufficient compressive strength to support construction loads.
3. Light enough in weight so that beams of commonly used lengths can be carried and placed by two men.
4. A support for tile spanning between beams.
5. A method of bonding so that span tile and concrete topping can be utilized in carrying compressive stresses.

The tile for the first beams were made by the Kalo Brick and Tile Company, Fort Dodge, Iowa, (fig. 6) by blocking off the upper corners of an old 5”x8”, three-cell, block die. The workers experienced no difficulty in handling the tile which were dried and burned with regular 8”x5”x12” blocks without loss in either process. The tile were de-aired, salt glazed and burned very...
Fig. 7. City hall and public library in Guthrie Center, Iowa. Tile beam construction was used.

hard. The weight per tile 12 inches long was 12½ pounds.

A slight warp was probably caused by friction against the attached plates and was easily eliminated in the later design.

The 4"x12"x24" span tile, run at the Redfield Brick and Tile Company plant at Redfield, Iowa, were cut with a regular 4"x12"x12" cutter with alternate wires removed. Although these tile were quite heavy and a little difficult to handle, there was practically no loss in drying or burning. The tile were run from a partition tile die, de-aired and burned to usual hardness for partition tile.

The utilization of the units developed in the floor construction is shown in fig. 5.

The beam tile were tested for compression by applying the load on the open end. Failure occurred at an average load of 70,000 pounds, giving a load of 5,250 pounds per square inch net area.

The 4"x12"x24" span tile for use between the beams were tested with a concentrated load in the center. The tile had a 2½" bearing at each support with the load applied on a 4" plate in the center. The average load at failure was 4,500 pounds.

Tension tests on the steel gave an average yield point of 42,300 pounds per square inch, and an average ultimate tensile strength of 65,500 pounds per square inch.

The mortar recommended for reinforced tile silos, consisting of 1 part cement, 3 parts sand and 1/3 part clay mortar mix, was used as the bonding medium in construction of the beams and sections. Mortar materials were screened to pass a No. 8 sieve, proportioned by volume and thoroughly mixed.
Water was added in sufficient quantity to produce a mix that worked well in "buttering" the ends of the beam tile. This consistency was used throughout the construction of the floor sections. Two-inch test cylinders of the mortar taken at intervals in the construction and cured with the beams failed at an average compressive stress of 1,835 pounds per square inch.

The beam tile were laid up end to end using a 2"x10" plank as a guide to produce a straight beam. The ends of the tile were "buttered," placed in position and tapped to get a tight joint. A 1/2" smooth, round reinforcing bar was dropped in the channel on each side of the beam. Sufficient mortar was squeezed up from the joints between the tile to keep the steel bars about 3/4 inch off the bottom. The mortar was then slushed into the channels, forced down around the bars with the trowel, leveled off and the joints pointed.

After 7 days the beams were carried and handled in a manner comparable to job conditions. Test sections (fig. 8) were made by placing 1/2-length span tile on each side of the beam comprising the section of floor which the beam would normally carry. The floor tile were dipped in water then placed on the channel of the beam after being well bedded down with mortar. Mortar was placed between the adjacent floor tile in the beam. The joints in the beam tile and floor tile were staggered. After the sections of floor tile were placed, the space above the beam was filled up to the level of the top of the flood tile and the entire section covered with 1 inch of concrete.
Beam and floor sections were covered with burlap and wetted down twice each day for 7 days. The age of the sections when tested ranged from 28 to 33 days.

TEST METHODS

One-hundred-pound bags of sand were used to load the sections, application of the loads being made at the third points to give an appreciable section subjected to uniform stress and to facilitate the making of observations (fig. 9).

On the bottom of the beams, at each point of support, a 1/4" steel plate, 4 inches in width, was embedded in plaster of paris to insure an even bearing surface. On the top of the section or beams, a similar plate was embedded at the third points.

The sections were supported on load-bearing tile covered with a steel plate. A steel roller was placed between the plate on the support and the plate on the beam to provide a freely loaded condition. A channel was placed on rollers resting on the plate at the third points to carry the applied load. One test was made on a beam applying the load at the third points on the channel part of the beam. All other tests on the beams without topping were made by applying the load on the top.

In the center of the sections the steel was exposed by chipping away tile and mortar on the bottom of the beams. Holes were drilled for the 8" strain gage. Brass buttons were set in the tile and mortar on the top of the beams and sections, and holes were drilled for gage readings.

An 8" Berry strain gage, equipped with an Ames dial

Fig. 9. A 10' beam with 3" floor tile carrying a load of 3,240 pounds.

Fig. 10. Tension failure of precast beam.
reading to 0.0002 of an inch was used to measure the elongation of the steel and the compression in the tile and concrete during the loading of the sections.

Deflection to 0.01 inch in the sections were read from a steel scale attached to a mirror. A wire was stretched from the neutral axis of the sections at the supports. The mirror and scale were mounted in the center of the beam.

The sections were set on the rollers on the supports and strain gage and deflections taken. The loads were applied in increments of 100 and 200 pounds, readings being taken after each loading.

In order to determine the recovery in the sections, various loads were placed and allowed to remain for periods of time from 2 to 24 hours. Readings were taken as each increment of load was removed from the section.

Two depths of sections were tested for shear by moving the points of application of the load to within 2 feet of the supports.

**LIVE LOAD—DEFLECTION PERFORMANCE OF SECTIONS**

The load-deflection results of the tests are given in table 1. Figure 12 shows the load-deflection curves for the sections with the load applied at the third points.

It is evident that the sections possess ample stiffness at design loads. In the beams tested without the floor tile and topping, the deflection was much greater but well below the allowable. The first sign of failure in the sections was nearly attained before the allowable deflection was obtained.

The 16' beam with 4" floor tile was loaded to 1½ times the allowable and left for 24 hours. When the load was removed, the instantaneous recovery was to within 0.09 inch of the original. The section was then loaded to the first sign of failure and the load removed. The permanent set was 0.48 inch.

The load-deflection performance of the 10' beam loaded at the ½ points and on the channel part of the beam was similar to that of the 10' beam loaded on the top.

**BENDING MOMENT—STEEL STRESS PERFORMANCE OF SECTIONS**

The experimentally determined stresses in the steel were obtained from the formula:
TABLE 1. DATA ON FLOOR TESTS—FIRST SERIES.

<table>
<thead>
<tr>
<th>Section</th>
<th>Beam length ft.</th>
<th>Tile depth in.</th>
<th>Span ft.</th>
<th>Width of sect. in.</th>
<th>Loads at failure</th>
<th>Max. bending moment in. lbs.</th>
<th>Stresses, lbs. per sq. in.</th>
<th>Design load</th>
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<tr>
<td></td>
<td>Applied Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>In steel</td>
<td>In compressive area</td>
<td>Lbs. per ft. factor (4)</td>
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<tr>
<td></td>
<td>Total</td>
<td></td>
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<tr>
<td>10</td>
<td>9.5</td>
<td>7.6</td>
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<td>12</td>
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<td>3655</td>
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<td>4,645</td>
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<td>4740</td>
<td>306</td>
<td>5,805</td>
<td>139,320</td>
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</table>
Fig. 12. Live load-deflection curves for the first series of beams.

\[ E_s = \frac{\text{unit stress in steel}}{\text{unit strain in steel}}, \]  
where \( E_s \), the modulus of elasticity, was taken as 29,000,000 lbs. sq. in.

The theoretically determined stresses plotted as straight lines were obtained from the formula:

\[ f_s = \frac{M}{A_s j d}, \]  
where

\( f_s \) = unit tensile stress in the steel in pounds per square inch.
\( M \) = external bending moment in inch-pounds.
\( A_s \) = area of steel in square inches.
\( j \) = ratio of the arm of the resisting couple to the effective depth.
d—effective depth.
(See page 254 for nomenclature.)

n, the ratio of the modulus of elasticity of the steel to the concrete, was determined from tests on these materials to be 15. Values of k and j were determined using this value.

The improved strength of the sections having the deeper floor tile is illustrated by comparing the stresses in the steel of the 16' beams with 4" and 5" span tile at the design bending moments. The bending moment of 36,000 inch-pounds produces a unit stress of 2,500 pounds per square inch in the 16' beam with 5" span tile. A bending moment of 28,000 inch-pounds produces a stress of 6,000 pounds per square inch in the 16' beam with 4" floor tile. The range between the experimentally and theoretically determined stresses is large for all the sections tested except the beams without span tile.

The improved strength of the sections having the deeper span tile was not so marked in the 14' beams with 3" and 4" span tile. There are two factors that tend to make the stresses more nearly the same than in the 16' beams with the 4" and 5" tile. First, the width of the 14' beam with 3" span tile is 28 inches, while the width of the 14' beam with 4" span tile is 16 inches. Second, the span tile in the 14' beam with 3" floor tile was a heavier tile. The same observations are true for the 10' beam with 3" and 4" span tile.

Figure 10 shows how the tile separated as the steel yielded. When the deflection reached 1.9 inches the vertical webs in the floor tile started cracking. Shortly after the cracking started the section collapsed.

**BENDING MOMENT—TILE AND MORTAR STRAIN**

All the beams tested without span tile and topping failed in compression (fig. 11). However, on the 10' beam shear cracks appeared below the points of application of the load running horizontally at the top of the channel just before the compression failure.

Using a value of E, the modulus of elasticity of the sections, of 2,000,000 pounds per square inch, the compressive stress in the 10' beam with 4" span tile was 151 pounds per square inch at the design bending moment of 31,200 inch-pounds. The compressive stress in the 10' beam with 3" span tile was 190 pounds per square inch at the design bending moment of 28,294 inch-pounds. The effect of the increased depth of floor can be noted by comparing the compressive stress in the 10' beam with 3" and 4" span tile for the same bending moment. With a 31,200 inch-pound moment, the stress in the top of the section with 4" tile was 151 pounds per square inch, while for the section with 3" tile the stress was 200 pounds per square inch. The effect is more noticeable
in the 16’ sections with 4” and 5” floor tile. With a bending moment of 36,630, the compressive stress is 300 pounds per square inch with the 4” tile and 200 pounds per square inch for the 5” tile.

The compressive stress in the 12’ section with 3” floor tile is 300 pounds per square inch at the design bending moment of 35,074 inch-pounds.

The calculated stresses as determined from the formula \( f_c = \frac{2M}{kjb^2} \) gave values quite close to the observed stresses using \( E_t = 2,000,000 \) pounds per square inch. For the 10’ beam with 3” floor tile at the design bending moment, the calculated stress was 187 pounds per square inch as compared to 180 pounds per square inch for the observed stress.

In the 16’ beam with 4” and 5” tile the calculated stresses were 214 and 216 pounds per square inch, respectively, at the design bending moment. The observed stresses were 200 and 180 pounds per square inch, respectively.

The observed stresses in the beams using \( E_t = 4,000,000 \) pounds per square inch were slightly higher than the calculated. For the 10’ beam the stress at the design bending moment was: Calculated, 1,380 pounds per square inch; observed, 1,830 pounds per square inch. For the 14’ beam the stress at the design bending moment was: Calculated, 1,090 pounds per square inch; observed, 1,140 pounds per square inch.

UNIT STRESSES DEVELOPED IN THE SECTIONS

Calculated stresses in sections at the maximum total load are given in table 1. This includes the dead load plus the maximum live load.

The values used in computing the stresses are given in table 2.

The entire width of section was used in computing the effective area. “n” was taken as 15 for the sections and 7.5 for the tile beams acting alone. In the sections the thickness of the tile in the beam and in the span tile was considered as an equal quantity of concrete. This practice was used in computing the effective compressive and shear areas. The stresses recorded have been calculated from the usual formulas

<table>
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<th>Beam section</th>
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<th>k</th>
<th>j</th>
<th>Effective area sq. in.</th>
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<td>5” floor tile</td>
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<td>.443</td>
<td>.852</td>
<td>33.25</td>
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</table>
of reinforced concrete design using the constants and factors given.

After the sections were tested to destruction, they were torn down to determine the amount of mortar that had run into the floor tile and the extent of bond between the mortar, beam tile and floor tile. The mortar ran into the 3" tile 1½ inches and into the 4" tile 2½ inches.

The bond between the mortar and the top of the beam tile was very poor when compared to the bond between the mortar and floor tile. The floor tile were well bonded to the channel of the beam.

**BEAM TESTS—SECOND SERIES (27)**

As a result of experiences gained in the first series of tests, numerous changes were made in the beam tile.

The modified design is shown in fig. 13. The increased depth was intended to give greater rigidity during construction. Although the volume of concrete above the beam tile is reduced, it is partially replaced by the increased thickness of the top of the beam tile. This increase in depth of beam tile results in more efficient use of the high compressive
strength of the top part of tile by placing it nearer the point of greatest stress.

Both beam and span tiles were scored with deep rectangular cuts, as shown on the sides of the beam tile, to assure a good bond with the concrete. If a smooth ceiling is desired the scoring should be restricted to unexposed faces.

In order to secure sufficient data to know what each floor section would do and to be able to design a floor system for
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<td>1.5</td>
<td>6,169</td>
<td>179,600</td>
<td>165</td>
</tr>
</tbody>
</table>

**Table 3. Floor Loading Test Data.**
given conditions of span and superimposed load, a group of beams were made, varying the following factors from probable minimum values to probable maximum values:

1. Length of span.
2. Spacing of beams (as fixed by the length of span tile).
3. Depth of beam (as fixed by the depth of span tile).
4. Depth of beam (as fixed by the thickness of concrete topping).
5. Diameter of reinforcing steel.

Span varied from 8 to 24 feet by 4' intervals; span tile varied from 12 to 24 inches. The span tile varied from 3 to 8 inches in depth. The steel varied from $\frac{1}{2}$ to 1" round rods, while the concrete topping thickness varied in $\frac{1}{2}$" intervals from $\frac{3}{4}$ to 2 inches.

The sections tested are illustrated in fig. 14 and described in table 3. Each consisted of a beam with one-half lengths of span tile on each side and with the customary concrete topping above. Sections 1, 2 and 3, comprising group A and tested before the schedule was adopted, were not considered in the comparative analysis. For this reason, further discussion on this group will be omitted here, and only groups B, C, D, E, F and G will be used. In each group only one of the five variables changed, and all other conditions were made identical as far as workmanship allowed.

### Table 4. Record of Test Samples—Crushing Strength of Concrete.

| Samples of concrete in forms surrounding steel. Cylinders 4" high, 2" diameter. Moist cured 7 days, remainder dry. (Area in sq. in. 3.1416) |
|---|---|---|
| No. | Age days | Load at failure, lbs. | Lbs. per sq. in. |
| 1. | 33 | 21,300 | 6780 |
| 2. | 33 | 14,620 | 4660 |
| 3. | 33 | 15,810 | 5030 |
| 4. | 33 | 15,180 | 4825 |
| 5. | 29 | 16,600 | 5280 |
| 6. | 29 | 18,730 | 4870 |
| 7. | 29 | 10,450 | 3350 |
| Average | 15,384 | 4896 |

| Samples of concrete topping. Cylinders 9" high, 4.5" diameter. Moist cured 7 days, remainder dry. (Area in sq. in. 15.90) |
|---|---|---|
| No. | Age days | Load at failure, lbs. | Lbs. per sq. in. |
| 11. | 30 | 75,850 | 4765 |
| 12. | 30 | 75,300 | 4730 |
| 13. | 30 | 43,450 | 2735 |
| 14. | 28 | 58,800 | 3700 |
| 15. | 28 | 76,790 | 4825 |
| 16. | 28 | 82,770 | 5200 |
| 17. | 28 | 76,790 | 4830 |
| 18. | 30 | 87,600 | 5500 |
| 19. | 30 | 80,200 | 5045 |
| 20. | 30 | 59,300 | 3730 |
| Average | 71,685 | 4506 |
Group G (section 23) was tested for elasticity. It was assumed to be a typical residential floor designed for 40 pounds per square foot. Its span of 16 feet was perhaps greater than the width of an ordinary room, but the longer span would require less loading and give larger deflection and deformation readings.

CONSTRUCTING TEST SECTIONS

Tile: The hollow tile were made by the Mason City Brick and Tile Company. All were de-aired except the 4”x6”x12” tile in section 9.

Steel: The steel consisted of deformed rods of intermediate grade.

Mortar:
1 part portland cement
3 parts sand
$\frac{1}{3}$ part clay mortar mix.

Concrete: Six gallons of water per sack of cement were used in making concrete for the topping.

The results of crushing tests of both the mortar and concrete topping are given in table 4.

Each beam tile was dipped in water, and the ends “buttered” as shown in fig. 15. These tile were laid end to end until the desired length of beam was obtained. Often a few taps with the hammer, as shown in fig. 16, were necessary to produce a joint of $\frac{1}{4}$ inch or less.
The continuous longitudinal channels of the beams were approximately half filled with concrete before the deformed steel rods were dropped into place. This rod was tapped down to within \( \frac{1}{4} \) to \( \frac{1}{2} \) inch from the bottom of the channel. More concrete was slushed into the channel, worked down with the trowel to insure a good bond and leveled off to finish the beams.

The finished beams were undisturbed for 7 days, during which period they were covered with wet burlap and sprinkled twice daily.

Before laying the half-span tile, the beams were sprinkled and the tile dipped in water to prevent the porous material from absorbing water from the mortar. Figure 17 shows a "buttered" span tile ready to be laid on the flange of beam bedded with mortar. The outer end of the half-span tile was not resting on an adjacent beam but temporarily rested on a 4"x4" timber properly elevated to make the span tile level as shown in fig. 18.

The space above the beam was then filled with concrete to a point level with the top of the filler tile (fig. 19). Little concrete ran into the 3" filler tile, while approximately 2½ inches of concrete ran into the 4" tile. This was sufficient to insure a good lock and anchorage and yet not excessive in use of concrete. To prevent a large amount of concrete from running into the 5", 6" and 8" tile, a drier mix was used. Shortly after this space was filled, the concrete topping was poured over the entire section and leveled. Concrete test samples were taken from approximately every third batch mixed. The sample cylinders, 4½ inches in diameter and 9 inches high, were made according to A. S. T. M. standards (1).
The sections were covered with wet burlap which was sprinkled twice daily for 7 days and then cured dry at approximately room temperature the 21 remaining days before testing. The samples were cured in a similar manner, 7 days wet and 21 days dry.
TESTING FLOOR SECTIONS

Pigs of iron and lead were used to load the section at the third points (fig. 20).

Under the beam, at each point of support, a steel plate \( \frac{3}{4}'' \times 4'' \times 8'' \) was firmly embedded in plaster of paris to insure a good bearing surface over which the reactions of the supports would be distributed. A similar steel plate was placed on the load-bearing tile supports. A steel roller, 2 inches in diameter, was placed between the two plates so that the sections would be freely supported.

On top of the sections, at each third point, a similar plate was embedded in plaster of paris. On each of these plates was placed another steel roller supporting a load-bearing tile and an oak plank upon which the weights were piled. Applying the load symmetrically at the third points produced a constant bending moment in the middle third of the span which was convenient for various measurements.

Deflection and concrete deformation readings were taken as described in the previous section. Steel deformation readings were taken in a similar fashion. Figure 22 shows the steel pegs, \( \frac{1}{2}'' \) inch round and \( \frac{3}{4}'' \) inch long, welded on the reinforcing rods, with holes drilled to fit the instrument. These pegs protruded nearly to the lower surface of the beam in

Fig. 22. Pegs welded on reinforcing bars for deformation readings.
openings cut into the tile before construction. Figure 23 illustrates the method of taking the steel deformation readings.

Deflection and deformation readings were taken after each loading increment of 4 pigs of iron or 2 pigs of lead equal to approximately 180 pounds.

The concrete and steel deformations were determined from the readings on the Berry strain gage. The total change in gage readings multiplied by 0.0002 gave the total deformation in the instrument length of 8 inches and when divided by 8 gave the unit strain caused by the live load applied.

The stresses in the steel were determined by the simple formula of multiplying unit strain by the modulus of elasticity. Tests were made on 16 steel rods taken from the ends of the floor sections to determine the modulus of elasticity. These rods were tested for tensile strength in a Southwark Emery testing machine, and deformation readings were taken at increment loadings of 1,000 pounds up to the elastic limit. The modulus of elasticity, $E$, was computed for each rod tested by the formula:

$$E = \frac{\text{unit stress}}{\text{unit strain}}.$$
The average E for the 16 rods tested was 27,240,000 pounds per square inch, and this figure was used to determine the steel stresses.
As concrete is not an isotropic material and will not conform to Hooke's law of elasticity, the slope of the stress-strain curve, interpreted as the modulus of elasticity, varies with the changing stresses.

Because of inherent difficulties, deformation readings were not taken on the 10 4½"x9" concrete test cylinders. The average crushing strength of these was 4,506 pounds per square inch. One of the specimens tested by the Iowa Highway Commission in a study of concrete deformation was almost identical with the 10 cylinders used in these tests, and the conditions of curing and testing were also similar. Accordingly, the modulus of elasticity calculated by the Iowa Highway Commission and shown in fig. 25 was used in determining the stresses in concrete.

**LIVE LOAD—DEFLECTION RELATIONSHIP**

The floor sections were very rigid, especially in the shorter spans, and safely carried loads several times the design load. The 8' span of section 4 deflected 0.23 inches under a live load of 12,192 pounds at the third points which was equivalent to 871 pounds per square foot. The allowable deflection of ⅛₀₀₀ of the span equals 0.267 inches. Up to this point the deflection was quite uniform as is shown by the live load deflection curve in fig. 26. Above 871 pounds per square foot, the deflection showed a marked increase, and the loading was continued to 13,728 pounds or 980 pounds per square foot. To prevent the section from falling and the weights from scattering and endangering helpers, the deflection was stopped by supports in the center after deflection reached satisfactory limits. In this way nearly all sections were removed whole.

Section 5 reached its allowable deflection of 0.40 inches under the live load of 320 pounds per square foot and finally failed at 364 pounds per square foot. Section 6, which was considered the typical floor for residential use, reached the allowable deflection of 0.533 inches at 149 pounds per square foot. This was equivalent to 3.725 times the design load of 40 pounds per square foot. Section 7 reached the allowable deflection of 0.667 inches at 80 pounds per square foot. Section 8 supported 40 pounds per square foot at the allowable deflection of 0.80 inches. Loading was continued until the 24' section had a total deflection of 14 inches or ⅛₀₀₀ of the span. At that point there were no signs of compression failure, and the steel continued to elongate until the section reached the ground. The large difference in deflection of group B was caused by the variation in the span and large
Fig. 26a. Live load—deflection curves.
GROUP C - VARIABLE LENGTH OF SPAN TILE (Span 16)

Fig. 26b. Live load—deflection curves.

GROUP D - VARIABLE DEPTH OF SPAN TILE (Span 16)

Fig. 26c. Live load—deflection curves.
GROUP E - VARIABLE STEEL (Span 16)

Fig. 26d. Live load—deflection curves.

GROUP F - VARIABLE CONCRETE TOPPING THICKNESS

Fig. 26e. Live load—deflection curves.
difference in total dead load of the sections. All sections in group B failed at approximately the same dead plus live load bending moment as is shown in table 4. From this standpoint, failure was very uniform, and the total dead plus live load was almost exactly inversely proportional to the span.

In most of the sections, the deflection limit was not reached until several times the design load was applied, as is shown in the live load-deflection curves. Every section except one failed by the elongation of the steel. For this reason the strength of the floor was almost directly proportional to the area of the steel or the effective lever arm $jd$ of the resisting moment. In general, the curves turned off slightly up to 50 pounds per square foot. Above 50 the curves had a tendency to approach a straight line but acted similar to stress-strain curves for concrete. At the point of failure they gradually turned off in a smooth, rounded fashion. Varying the thickness of concrete topping in group F had no great influence on the strength of the floor, and the extra thickness does not seem to justify its added cost.

Figure 27 shows section 14 failing under 7,297 pounds. Failure was considered to be the point where the deflection increase per loading increment was decidedly greater than the deflection increase produced by similar past loading increments. Table 4 shows the deflection at the load previous to the one causing the section to fail. The first load causing an increased rate of deflection was considered as the ultimate load at the third points. This was changed into the equivalent uniform live load by multiplying by $\frac{3}{2}$, derived from
the ratio of the two bending moment formulas $M=WL/6$ for third point loading and $M=WL/8$ for uniform loading. By assuming a factor of safety of 4, the design live load per square foot was computed from the ultimate live load per square foot. The deflection reading at the design load was, in most cases, only one-fourth or one-fifth of the allowable, considered at $1/360$ of the span. The deflection under the maximum load was approximately $1\frac{1}{2}$ times the allowable, assuring ample rigidity.

**LIVE LOAD—CONCRETE DEFORMATION RELATIONSHIP**

Figure 28 gives the live load—concrete deformation relationship of the various sections tested. In nearly all cases, the deformation curve had a tendency to follow a straight line. In a few instances a reverse curve started near the halfway mark to failure. The curve for section 9 was slightly higher than 10, 11 and 6, due perhaps to the fact that the span tile were not de-aired. The only section which failed to follow its theoretical path was 18. No reason for its action is given unless the readings were not taken accurately, although the instrument was checked with the standard bear several times during each testing. Some difficulty in plotting results from the fact that loading increments were so small that resulting strain in the concrete could not be measured. Hence, the curves show several loadings with no increase in strain.

Some points of section 19 did not fall on a smooth curve, although later points show that they had a tendency to follow such a path. The concrete was understressed in all cases, for in only a few observations did it exceed 2,000 pounds per square inch and was capable of taking over 4,000. Not a single section in this series failed in compression even though the neutral axis was moved up considerably in large deflections. It was thought that section 19 would fail in compression, but instead the large rods were merely pulled in the concrete surrounding them before the elastic limit was reached.

**LIVE LOAD—STEEL DEFORMATION RELATIONSHIP**

The failure of every section tested, except one, was caused by the elongation of the steel rods. The steel allowed the beam to deflect slowly and at the ultimate load slowly elongated. In only one instance did the section collapse suddenly. The steel deformed in fairly uniform increments, as indicated by the curves shown in fig. 29. Observations on the steel were discontinued when the load was such as to make the taking of such observations hazardous.
Fig. 28a. Live load—concrete deformation curves.
GROUP C - VARIABLE LENGTH OF SPAN TILE (Span 16)

Fig. 28b. Live load—concrete deformation curves.

GROUP D - VARIABLE DEPTH OF SPAN TILE (Span 16)

Fig. 28c. Live load—concrete deformation curves.
Fig. 28d. Live load—concrete deformation curves.

Fig. 28e. Live load—concrete deformation curves.
GROUP B - VARIABLE SPAN

Fig. 29a. Live load—steel deformation curves.
GROUP C - VARIABLE LENGTH OF SPAN TILE (Span 16)

Fig. 29b. Live load—steel deformation curves.

GROUP D - VARIABLE DEPTH OF SPAN TILE (Span 16)

Fig. 29c. Live load—steel deformation curves.
GROUP E – VARIABLE STEEL (Span 16)

Fig. 29d. Live load—steel deformation curves.

GROUP F – VARIABLE CONCRETE TOPPING THICKNESS (Span 16)

Fig. 29e. Live load—steel deformation curves.
The middle third of the span of every section showed several cracks in the beam as illustrated in fig. 30. Not only did the joints open, but the beam tile showed hair-line cracks gradually widening as the load increased. In cases of excessive deflection the span tile cracked also, while the joints readily showed the upward movement of the neutral axis. Figure 31 shows one of the earlier sections with the crack wide enough to expose the steel rods and enough deflection to move the neutral axis high enough to cause the concrete to crumble.

Section 19 was the only section to fail in bond. The deflection, concrete and steel deformation increased uniformly until the applied live load reached 12,204 pounds, when, suddenly, the entire section and load dropped to the floor as shown in fig. 32. It is quite evident in fig. 33 that the steel bars pulled in the concrete. The 1" round rod was too large for the channel in the beam tile and did not allow sufficient concrete to insure good bond. From samples taken,
cracks were so noticeable that the wedge action of the pulling deformed bar forced the channels of the beam to open and allowed the rod to slip.

**TESTING FOR ELASTICITY**

The floor is also very elastic. A section 16 feet long with 24" filler tile of 4" depth, two 5/8" round rods and a 1½" concrete topping was tested for the properties of elasticity. The section was loaded three times to a load equivalent to 80 pounds per square foot or twice the assumed design load of 40 pounds per square foot used in residential design. The section was then loaded to three times the design load or 120 pounds per square foot and the next time to 164 pounds per square foot or four times the design load. After each loading and unloading, a 5-minute rest period was allowed for recovery in case there should be such. The results are shown in fig. 34.

The first loading produced a deflection of 0.25 inches or less than one-half of the allowable of 0.533, assuming \( \frac{1}{360} \) of the span. When all but 200 pounds (7 pounds per square foot) were removed, the section took a set of 0.07 inches. The deflection coincided identically with the first at both 7 and 80 pounds per square foot during the next three load-
ings and unloadings. Each succeeding curve had a greater tendency to approach a straight line than the one before it. At 120 pounds per square foot the deflection read 0.42 inches after taking a drop of 0.02 inches during the 5-minute period. When unloaded the deflection set was 0.04 inches more than before, or a total of 0.11 inches.

The allowable deflection was reached at 143 pounds per square foot, but loading was continued up to 164, at which point the section deflected 0.01 inches during the 5-minute period for a total of 0.65 inches. When unloaded the deflection read 0.14 inches and failed to recover further during the next 24 hours.
The sixth loading was primarily to determine the effect of time on the loaded section. Eighty pounds per square foot produced an increased deflection of 0.01 inches, while at 124 pounds per square foot the deflection increased 0.03 inches during the same length of time. The load of 163 pounds per square foot remained on the section for several days. The first 24-hour period produced 0.03 inches deflection, the second
day 0.02 inches, and each day's deflection thereafter for 10 days increased 0.01 inches. The next 10 days the daily increase averaged 0.005 inches.

The concrete deformation shown in fig. 35 behaved in a manner similar to the deflection. In nearly every case the live load-deformation curve was a straight line with a lag following the loadings and unloadings. After the 5-minute period, the deformation had a tendency to assume its original position after the first loading and at similar loads reached the same points.

The steel deformation shown in fig. 36 showed a gradual tendency to increase with the number of loadings. At each successive loading the live load-deformation curve moved to the right, and in only two cases did it come back to the same point when unloaded. The steel deformation curves are, as a rule, straight and did not deviate from this course during the 24-hour periods on the sixth loading.

CALCULATION OF STRESSES

Two basic assumptions in the theory of flexure are:
1. That the deformations of the fibres of the entire effective cross section under stress vary directly as the distance from the neutral axis.
2. That for either material, steel or concrete, unit stress is proportional to deformation.

It is commonly known that in the case of concrete, stress is not directly proportional to deformation; but it has become common practice to so regard it in reinforced concrete design or, in other words, to assume "straight line" variation.

These two assumptions were adopted in the calculation of the extreme fibre stresses of the sections tested.

Table 5 gives the comparative results of observed and calculated stresses produced by the maximum live load the section safely supported.

To compute the extreme fibre stresses, reinforced concrete T beam formulas were applied.

The observed stresses in the steel and concrete checked closely with the calculated.

Using the transformed section method, it was discovered that the neutral axis may fall either in the stem or the compressive flange, depending upon the proportions of the beam. The compression area was assumed to be the distance center to center of supporting beams and consisting of the full layer of concrete and the upper ½ inch of span tile.

Using:

n, the ratio of the modulus of elasticity of steel and concrete as determined from tests to be 10,
d, the effective depth from the top of the concrete layer to the center of the steel,
k, the distance from the top to the neutral axis, and t, the depth of the compression flange, k was determined

CASE I

(Neutral Axis in Compression Flange)

To obtain the value of j, the ratio of the lever arm of the couple which forms the resisting moment of the beam, to
<table>
<thead>
<tr>
<th>No.</th>
<th>k</th>
<th>d</th>
<th>j</th>
<th>A_s</th>
<th>j_d</th>
<th>Steel stress, lb, per sq. in.</th>
<th>Bending moment, lb-in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>0.219</td>
<td>7.10</td>
<td>1.55</td>
<td>0.927</td>
<td>7.10</td>
<td>1.890</td>
<td>1,247</td>
</tr>
<tr>
<td>5</td>
<td>0.219</td>
<td>7.10</td>
<td>1.55</td>
<td>0.927</td>
<td>7.10</td>
<td>1.900</td>
<td>1,247</td>
</tr>
<tr>
<td>6</td>
<td>0.219</td>
<td>7.10</td>
<td>1.55</td>
<td>0.927</td>
<td>7.10</td>
<td>1.900</td>
<td>1,247</td>
</tr>
<tr>
<td>7</td>
<td>0.219</td>
<td>7.10</td>
<td>1.55</td>
<td>0.927</td>
<td>7.10</td>
<td>1.900</td>
<td>1,247</td>
</tr>
<tr>
<td>8</td>
<td>0.219</td>
<td>7.10</td>
<td>1.55</td>
<td>0.927</td>
<td>7.10</td>
<td>1.900</td>
<td>1,247</td>
</tr>
<tr>
<td>9</td>
<td>0.278</td>
<td>7.10</td>
<td>1.76</td>
<td>0.987</td>
<td>7.10</td>
<td>1.815</td>
<td>1,460</td>
</tr>
<tr>
<td>10</td>
<td>0.251</td>
<td>7.10</td>
<td>1.79</td>
<td>0.917</td>
<td>7.10</td>
<td>1.640</td>
<td>1,460</td>
</tr>
<tr>
<td>11</td>
<td>0.233</td>
<td>7.10</td>
<td>1.66</td>
<td>0.923</td>
<td>7.10</td>
<td>1.640</td>
<td>1,460</td>
</tr>
<tr>
<td>12</td>
<td>0.239</td>
<td>7.10</td>
<td>1.46</td>
<td>0.920</td>
<td>7.10</td>
<td>1.640</td>
<td>1,460</td>
</tr>
<tr>
<td>13</td>
<td>0.210</td>
<td>7.10</td>
<td>1.70</td>
<td>0.930</td>
<td>7.10</td>
<td>1.640</td>
<td>1,460</td>
</tr>
<tr>
<td>14</td>
<td>0.200</td>
<td>7.10</td>
<td>1.82</td>
<td>0.933</td>
<td>7.10</td>
<td>1.640</td>
<td>1,460</td>
</tr>
<tr>
<td>15</td>
<td>0.223</td>
<td>7.10</td>
<td>1.82</td>
<td>0.933</td>
<td>7.10</td>
<td>1.640</td>
<td>1,460</td>
</tr>
<tr>
<td>16</td>
<td>0.182</td>
<td>7.10</td>
<td>1.82</td>
<td>0.933</td>
<td>7.10</td>
<td>1.640</td>
<td>1,460</td>
</tr>
<tr>
<td>17</td>
<td>0.219</td>
<td>7.10</td>
<td>1.15</td>
<td>0.927</td>
<td>7.10</td>
<td>1.815</td>
<td>1,460</td>
</tr>
<tr>
<td>18</td>
<td>0.219</td>
<td>7.10</td>
<td>1.15</td>
<td>0.927</td>
<td>7.10</td>
<td>1.815</td>
<td>1,460</td>
</tr>
<tr>
<td>19</td>
<td>0.219</td>
<td>7.10</td>
<td>1.15</td>
<td>0.927</td>
<td>7.10</td>
<td>1.815</td>
<td>1,460</td>
</tr>
<tr>
<td>20</td>
<td>0.219</td>
<td>7.10</td>
<td>1.15</td>
<td>0.927</td>
<td>7.10</td>
<td>1.815</td>
<td>1,460</td>
</tr>
<tr>
<td>21</td>
<td>0.219</td>
<td>7.10</td>
<td>1.15</td>
<td>0.927</td>
<td>7.10</td>
<td>1.815</td>
<td>1,460</td>
</tr>
<tr>
<td>22</td>
<td>0.219</td>
<td>7.10</td>
<td>1.15</td>
<td>0.927</td>
<td>7.10</td>
<td>1,815</td>
<td>1,460</td>
</tr>
</tbody>
</table>
the effective depth, d, the following formulas were used:

- \[ k = \frac{\sqrt{2pn + pn^2} - pn}{n} \]
- \[ j = 1 - \frac{k}{3} \]
- \[ p = \frac{2r(n + r)}{2r(n + r)} \]
- \[ p = \frac{A_s}{bd} \]

where \( p \) = ratio of effective area of tensile reinforcement to effective area of concrete in the beams.

The following formulas are simply the valuation of the couple, total compression times lever arm, and that of the same couple expressed in terms of unit stress in steel:

- \[ M_c = \frac{1}{2} f_c \frac{k}{j} bd^2 \]
- \[ M_s = A_s f_s j d \]

where

- \( f_c = \) extreme unit fibre stress in concrete
- \( f_s = \) extreme unit fibre stress in steel

CASE II

(Neutral Axis Below Compression Flange)

To obtain the value of j, the ratio of the lever arm of the couple which forms the resisting moment of the beam, to the effective depth, d, the following formulas were used:

- \[ jd = d - z \]
- \[ z = \frac{3kd - 2t}{2kd - t} \cdot \frac{t}{3} \]
- \[ k = \frac{np + \frac{1}{2} (t/d)^2}{np + \frac{t}{d}} \]

where \( z \) is simply an expression for the location of the center of gravity of the trapezoidal compression area.

The following formulas are simply the valuation of the couple, total compression times lever arm, and that of the same couple expressed in terms of the unit stress in steel:

- \[ M = f_c \left(1 - \frac{t}{2kd}\right) btjd \]
- \[ M = A_s f_s j d \]

where

- \( f_c = \) extreme unit fibre stress in concrete
- \( f_s = \) extreme unit fibre stress in steel

and

- \( A_s = \) area of steel

The bending moment in table 5 is that produced by the live load at a point where the last deformation reading was taken. This was usually the last load the section safely supported.
DEMONSTRATION FLOORS
DES MOINES, IOWA

In addition to the tests made at Ames, several performance tests were made in other localities. The first one was made in a Des Moines residence (figs. 37, 38) at the request of the contractor and under the close supervision of the city building inspector.

The 12' span consisted of beams, with two 5/8" rods, placed 16 inches on center using the 12" filler tile and 1 1/2 inches of concrete topping. The floor had been placed only 8 days. An area 9 feet wide, covering 6 beams, was uniformly loaded to the design load of 40 pounds per square foot with sacks of cement. An Ames dial gage, securely set up and braced under one of the middle beams at the center of the span, showed a deflection under the 40-pound loading of 0.009 inches. An additional 40 pounds per square foot produced a total

Fig. 37. Loading demonstration floor in Des Moines.

Fig. 38. Apparatus for measuring deflection of demonstration floor in Des Moines.
deflection of 0.018 inches. After a 24-hour period, the deflection increased to 0.025 inches. This is materially under the .400 inches which were allowable under the Des Moines building code which specifies a limit of $\frac{1}{400}$ of the span under twice the design load.

LINCOLN, NEBRASKA

A somewhat similar test was conducted by the Clay Products Institute for and under the supervision of D. L. Erickson, City Engineer, Lincoln, Neb., on Nov. 13, 1935.

The section tested consisted of three beams 14' 3" in length and spaced 28 inches o. c. The tile used in constructing the beams were 8 inches wide at the bottom, 3 inches wide at the top, 12 inches long and 6 inches deep. The floor tile used in spanning between the beams were 12 inches wide, 24 inches long and 4 inches deep.

MATERIALS USED

Steel—$\frac{5}{8}$" round, deformed, intermediate grade reinforcing bars.

Concrete—1 part portland cement
4 parts pit-run sand and gravel

Mortar—1 part portland cement
3 parts clean sand
$\frac{1}{8}$ part plasticizer

PROCEDURE IN BUILDING THE TEST SECTION

The upper half of the beam tile were “buttered” with mortar and placed end to end on a plank. The reinforcing bars were then placed and the channels filled with concrete.

The beams were sprinkled once a day for 3 days after building. When the beams were 5 days old they were carried to and set on a wall section, each beam having a 4" bearing on the walls. After the beams were placed, a bed joint was carried along the top of the channel and the span tile set. The joints between the span tile were filled with mortar.

A 1½" layer of concrete was then placed over the entire floor. The section was sprinkled for 5 days and was tested in 14 days.

The live load was distributed uniformly and placed in increments of 600 pounds, 100-pound sacks of sand being used.

This floor was designed for 50 pounds per square foot and supported nearly six times the required live load. The allowable deflection of $\frac{1}{400}$ of the span or 0.533 inches was not reached until 192 pounds per square foot or nearly four times the design load was applied.

The load-deflection performance of the section is given in table 6. The failure under 293 pounds per square foot was caused by the elongation of the steel rods.
TABLE 6. LOAD-DEFLECTION PERFORMANCE.

<table>
<thead>
<tr>
<th>Total load (lbs.)</th>
<th>Load per sq. ft. (lbs.)</th>
<th>Deflection (ins.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4,275</td>
<td>1,500</td>
<td>45</td>
</tr>
<tr>
<td>4,275</td>
<td>5,000</td>
<td>45</td>
</tr>
<tr>
<td>4,275</td>
<td>7,400</td>
<td>45</td>
</tr>
<tr>
<td>4,275</td>
<td>11,000</td>
<td>45</td>
</tr>
<tr>
<td>4,275</td>
<td>14,000</td>
<td>45</td>
</tr>
<tr>
<td>4,275</td>
<td>17,000</td>
<td>45</td>
</tr>
<tr>
<td>4,275</td>
<td>19,400</td>
<td>45</td>
</tr>
<tr>
<td>4,275</td>
<td>27,700</td>
<td>45</td>
</tr>
</tbody>
</table>

IOWA CITY, IOWA

The following is abstracted from a report dated Jan. 24, 1938, by Edward Soucek, registered engineer, describing a test made by him on the floor of the drill hall of the armory built for the Armory Building Corporation of Iowa City, Iowa. This floor was designed for a live load of 125 pounds per square foot. A test load of 250 pounds per square foot, twice the design load, was applied as indicated in fig. 39. City specifications required that the deflection after 24 hours under the above loads should not exceed $\frac{1}{360}$ of the span and without the occurrence of cracks or other indications of failure. The test was made Dec. 23 to 26, 1937, about 6 or 7
weeks after the slab was poured. Available materials, tar in barrels, maple flooring and cement in bags, were used for the live load. On panel "A" 24,010 pounds were placed and 23,860 on panel "B" or a total of 47,870. Deflections were read from below the floor on a graduated rod with a steel point, slow motion screw and vernier reading to 1/1000 feet.

The results of the test on panel "A" are tabulated below:

<table>
<thead>
<tr>
<th>Total load</th>
<th>Deflection (ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6,000</td>
<td>0.001</td>
</tr>
<tr>
<td>11,440</td>
<td>0.002</td>
</tr>
<tr>
<td>17,690</td>
<td>0.004</td>
</tr>
<tr>
<td>24,010</td>
<td>0.006</td>
</tr>
</tbody>
</table>

The ultimate deflection of panel "A" after the above loads had remained in place for about 48 hours and panel "B" had been fully loaded for about 24 hours was 0.009 feet.

The ultimate deflection under test load and the permanent deflections after the loads were removed are given below:

<table>
<thead>
<tr>
<th>Panel</th>
<th>Deflection under test load (ft.)</th>
<th>Permissible</th>
<th>Deflection percent of permissible</th>
<th>Permanent set</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.009</td>
<td>0.041</td>
<td>22</td>
<td>0.008</td>
</tr>
<tr>
<td>B</td>
<td>0.013</td>
<td>0.041</td>
<td>32</td>
<td>0.006</td>
</tr>
</tbody>
</table>

The deflection on panel "B" was measured with a surveyor's level.

"The girder deflections were not measured separately so the girder deflections are included in all observations.

"No cracks appeared in the slab at any time during the tests. Spaces were left in the load in the region of maximum negative bending moment to allow inspection, so if any cracks visually perceptible had occurred, they would have been observed.

"Since the panels tested showed deflections less than one-third of those permissible and developed no signs of cracking, it was concluded that the floor was safe for the design load.

"The width of the loaded area was 2.8 times the spacing of the tile beams. The joist immediately under the load in tests covering a limited area receive some assistance from adjacent beams, the extent of which participation could be reduced by loading a larger area."

LEAD, SOUTH DAKOTA

A section of floor was tested at Lead, S. D., (fig. 40) for PWA Engineers before this construction was installed in the new city hall. The section tested was five joists wide with a joist space of 16 inches on center. The span was 12 feet.
Fig. 40. Demonstration floor, Lead, S. D.

6 inches; the design load 200 pounds per square foot. The section was tested for concentrated loads and uniform loads. The section was loaded to 720 pounds per square foot and allowed to stand for 6 days. The increase in deflection was only \( \frac{1}{2} \) of an inch. The section finally failed when loaded to 885 pounds per square foot.

TYPICAL INSTALLATIONS

The versatility of the precast tile beam floor is well illustrated in the photographs of completed jobs. By the use of long-span tile or filler blocks, the beams can be spaced at greater intervals and the relative cost reduced, thus adapting its use to low-cost residential construction. Slight modifications such as shortening and deepening the span tile, provide a much stronger structure quite adequate for heavier loads.

These properties have resulted in wide acceptance by architects, engineers and building inspectors in the Midwest. There have been hundreds of jobs installed in many types of buildings from low-cost residences to hospitals and schools. The floor has been used extensively in Omaha, Lincoln, Des Moines, Mason City, Iowa City, Burlington, Marshalltown,
Minneapolis and St. Paul and many other cities in the Midwest.

The Federal Housing Administration has approved this construction in houses submitted to them for insured loans.

The wide usage of the floor is illustrated by the variety of jobs in the partial list of installations given below.

PUBLIC BUILDINGS

City Hall—Guthrie Center, Iowa
  Wm. Nielsen, Architect, Des Moines, Iowa
City Hall—Lead, S. D.
  R. S. Fraser & Don Smith, Architects, Lead, S. D.
Robertson School—Robertson, Iowa
  George Meyers, Designer
Armory—Iowa City, Iowa
  Fiske & Ruth, Architects, Iowa City, Iowa
Bath House—Springfield, Minn.
American Legion Memorial Pool
N. Y. A. Development—Stillwater, Okla.
  Philip A. Wilber, Architect
Church—Avalon, Iowa
School—Ira, Iowa
  Wm. Nielsen, Architect, Des Moines, Iowa
Dormitory—Stillwater, Okla.
  Philip A. Wilber, Architect, Stillwater, Okla.
N. Y. A. Development (2 buildings), Stillwater, Okla.
  Philip A. Wilber, Architect, Stillwater, Okla.
RESIDENCES AND APARTMENTS

Windsor Terrace Apartments, Des Moines, Iowa
   Proudfoot, Rawson, Brooks & Borg, Architects
Alpha Gamma Rho—Stillwater, Okla.
   Byron Miller, Designer
Beta Theta Pi—Stillwater, Okla.
   Philip A. Wilber, Architect
General Electric Model Home—Omaha, Neb.
   Oscar T. Bowles, Architect, Omaha, Neb.
M. D. Judd Residence—Mason City, Iowa
A. C. Frisk Residence—Mason City, Iowa
   Hansen & Waggoner, Architects, Mason City, Iowa
W. J. Goodwin, Jr., Residence—Des Moines, Iowa
   Kraetsch & Kraetsch, Architects, Des Moines, Iowa
Frank Warren Residence—Minneapolis, Minn.
   Carl Gage, Architect, Minneapolis, Minn.
John Cullen Residence—Lincoln, Neb.
   Davis & Wilson, Architects, Lincoln, Neb.
Apartment—Omaha, Neb.
   Frank Reida, Architect, Omaha, Neb.
Residence 1949 E. River Road—Minneapolis, Minn.
   Ellerbe & Company, Architects, St. Paul, Minn.
M. J. Haaheim Residence—Mason City, Iowa
   M. J. Haaheim, Builder
Chittenden Residence—Burlington, Iowa
   H. S. Muesse, Architect, Davenport, Iowa
Paul Goodwin, Residence—Des Moines, Iowa
Dan Guy Residence—2½ miles north, Burlington, Iowa
John Swanson Residence—Minneapolis, Minn.
   Carl Gage, Architect, Minneapolis, Minn.
HOSPITALS AND CLINICS

Tracy Hospital—Tracy, Minn.
Don Parsons, Architect

Junior League Hospital—Des Moines, Iowa
John Normile, Architect, 511 Hubbell Building, Des Moines, Iowa

Clinic—Estherville, Iowa
Thorwald Thorson, Architect, Forest City, Iowa

Municipal Hospital, Ada, Minn.

Victory Hospital—Robbinsdale, Minn.
Ed Malm, Architect

Benson Medical Center, Omaha, Neb.
Oscar T. Bowles, Architect, Omaha, Neb.

Kerr Clinic and Apartment—Ames, Iowa
Oscar Woody, Architect

Marshall County Home—Marshalltown, Iowa
Russell Prescott, Architect

COMMERICAL BUILDINGS

Roberts Dairy—Lincoln, Neb.
Meginnis & Schaumbert, Architects, Lincoln, Neb.

Barber Shop, Oelwein, Iowa
John Lippert, Builder

Liquor Store, Ackley, Iowa
George Meyer, Builder
Liquor Store—Minneapolis, Minn.
  Carl Gage, Architect, Minneapolis, Minn.
Liquor Store—Traer, Iowa
  Charles Seda, Builder
Farmers’ Mutual Telephone Building—Rockford, Iowa
Garage and Service Station—Manning, Iowa
  Farmers Union Service Association
Brewster Dairy—Washington, Iowa
  J. E. Kupka, Builder
Johnston’s Greenhouse—Mason City, Iowa
  Henry Tageson, Builder
Blacksmith Shop—Carroll, Iowa
  J. Howard Hodges, Builder
Electric Store and Apartments—Mason City, Iowa
  H. C. Determan, Owner
  Karl Ahlman, Builder
Warren Brothers—Chevrolet garage—Ivy, Iowa
Filling Station—Weich Brothers—Reinbeck, Iowa
Office Additions, Kellogg Milling Company, Des Moines, Iowa
  Bert Hokel, Builder

WAREHOUSES AND INDUSTRIAL BUILDINGS

Cement Storage—Iowa City, Iowa
  Hawkeye Lumber Company
Warehouse, Mill & Shops—Deadwood, S. D.
  Fish and Hunter Company; Don Smith, Architect, Deadwood, S. D.
Hides and Wool Storage—Marshalltown, Iowa
  Joseph Krantman, Owner
Lumber Yard and Offices—Forest City, Mo.
Hilleman Packing Plant—Marshalltown, Iowa
DESIGN

PRECAST CONSTRUCTION

When tile joists are precast in the usual way, placed in position, span tile placed and top slab poured, the structural design is made in accordance with the mechanical principles that apply to a reinforced concrete “T beam.” For convenience the customary nomenclature is given below:

 NOMENCLATURE

\[ b = \text{width of flange of T beam} \]
\[ b' = \text{thickness of web of T section} \]
\[ d = \text{depth from compression face of slab to center of longitudinal tensile reinforcement} \]
\[ E_c = \text{Modulus of elasticity of concrete in compression} \]
\[ E_t = \text{Modulus of elasticity of tile in compression} \]
\[ E_s = \text{Modulus of elasticity of steel in tension or compression} \]
\[ f_c = \text{Unit compressive stress in extreme fibre of concrete in flexure} \]
\[ f'_{c} = \text{Ultimate compressive strength of concrete (usually at age of 28 days)} \]
\[ f_s = \text{Unit tensile stress in longitudinal reinforcement} \]
\[ j = \text{Ratio of distance between centroid of compression and centroid of tension to the depth d} \]
\[ k = \text{Ratio of distance between the extreme fibre and neutral axis, to the effective depth or total depth} \]
\[ L = \text{Span length of beam or slab} \]
\[ M_o = \text{Bending moment or moment of resistance in general} \]
\[ n = \text{Ratio of modulus of elasticity of steel to that of concrete} \]
\[ \Sigma = \text{Sum of perimeters of bars} \]
\[ p = \text{Ratio of effective area of tensile reinforcement to effective area of concrete in beams} \]
\[ s = \text{Spacing of stirrups in a direction parallel to longitudinal reinforcement} \]
\[ t = \text{Thickness of the flange of T beams} \]
\[ u = \text{Bond stress per unit of surface area of bar} \]
\[ v = \text{Unit shearing stress} \]
V = Total shear  
\( w = \) Uniformly distributed load per unit of length of beam  
\( W = \) Total dead and live load uniformly distributed.

Flexure formulae used in the design of precast tile joist floors are given here for reference:

Case II, when \( kd > t \)

\[
r = \frac{f_s}{f_c} \quad p = \frac{n}{2r (n+r)}
\]

\[
f_c = \left( 1 - \frac{t}{2kd} \right) btjd
\]

\[
f_s = \frac{M_s}{A_s jd}
\]

Note: To find values \( k \) and \( j \), refer to p. 267, "Reinforced Concrete," by Caughey, (10) or other textbooks on reinforced concrete.

a. Flexural Computations. It is assumed for the majority of cases that joists are simply supported. To determine the maximum positive bending moment for simple beams, the customary formula, \( WL/8 \), is employed. In the case of continuous equal spans, the same formula applies. No reduction is made in the coefficient because true continuity cannot exist. However, some continuity will develop when slab is poured in one operation over both spans. For this reason negative steel should be provided at the interior support to check cracking that would be caused by tension in the top slab at that point. The customary negative moment coefficient should be used for selecting reinforcement for the particular loading and supporting system encountered.

In the case of continuous beams with unequal spans, maximum bending moments may be determined by the formula \( WL/8 \) for dead loads only, combining these with moments determined by a continuity analysis of the problem using the live load only.

b. Reinforcement for bending. Steel placed in the channels of the joists during casting provides the longitudinal tensile reinforcement for positive bending moment. The area of steel required to resist maximum positive bending moment is determined by the formula

\[
A_s = \frac{BM_{max}}{f_s jd}
\]

For practical consideration of bond, bars larger than \( \frac{7}{8} \)" round should not be used in the regular 6"x8" size joist.

c. Computation of shear. The formula used for determining the unit of shearing stress is

\[
v = V / b'jd
\]
For regular 6"x8" tile joists, the width, $b'$, is taken as 6 inches when span tile are set on mortar and 5 inches with drop filler tile set dry; for the 8"x10" tile joists, the width, $b'$, is taken as 8 inches.

d. Provision for shear. If the maximum unit shearing stress exceeds the allowable, as set up in the A. C. I. code (1), stirrups may be provided, if it is decided not to decrease the amount of joist spacing; or, the hollow portion of the joist may be filled with concrete for the required distance from the support. Filling the joists increases $b'$ to 8 inches. Special anchorage (see A. C. I. code) may be used to increase allowable unit shear stress.

e. Computation of bond. Bond stress between reinforcing steel and concrete is determined by the formula 

$$ u = \frac{V}{E_{oj}d} $$

Refer to the A. C. I. code (1) for allowable bond stresses.

f. Bond and anchorage provisions. By providing special anchorage of longitudinal steel as specified in the A. C. I. code, (1) allowable bond and shear stresses are increased. In general, anchorage requirements of the A. C. I. code may be followed.

g. Outline of design procedure. 1. Select joist spacing for trial. For economy, wherever possible the widest joist spacing should be used. 2. Determine maximum bending moment caused by dead and assumed live loads. 3. Provide tensile reinforcement. 4. Check compression in concrete caused by flexure. 5. Check shear, bond and anchorage and make any necessary provisions. 6. Check compliance of design with A. C. I. code. Computations for a typical design are shown in fig. 46.

BUILT-IN-PLACE CONSTRUCTION

(Top slab is poured integrally with joist concrete surrounding longitudinal steel.) Where long joist spans are required, it may be more economical and easier to construct the floor
by building forms and setting the joist tile and steel in place upon them. Since the precast joists weigh about 21 pounds per lineal foot, lengths 20 feet and more present a handling problem because of their weight. When joist tile, reinforcing steel and span tile are in place, all concrete is poured in one operation, including the concrete that surrounds the steel
in the channels of the tile joist. Large joist tile, 8"x10"x12" are used, and bars up to 1½" square may be used.

Simple spans are designed as usual. However, continuous spans are designed according to the principles of continuity. Longitudinal steel may be bent-up and continued over supports to resist negative bending moment as is customary. Maximum bending moments may be determined by moment distribution, slope-deflection, conjugate points, theorem of three moments or any recognized correct method (10). Reference should be made to the A. C. I. code (or governing code) for guidance in the design, making the proper provisions for flexure, shear, bond, etc.

USING JOIST DESIGN TABLES

To use the present tile joist design tables it is first necessary to know what loads, other than the weight of the floor itself, the floor will be expected to carry. These loads are commonly called “superimposed loads” or “live loads.” The simplest method of explaining the use of curves is by example.

Assume that it is desired to determine the spacing of joists and size of reinforcing steel necessary for a precast tile joist floor in a residence. The joists in question span the recreation room in the basement and form the floor of the living room on the first floor. The clear span is 15' 0". In other words, the total distance between the inside faces of the exterior foundation wall and the interior bearing wall (or in some cases the inside face or edge of the steel or concrete girder) is 15' 0". The owner desires a suspended ceiling in the basement recreation room in order to conceal all pipes and heating ducts. The living room floor will have a pad and carpet on it.

Now to determine the superimposed load: The live load required for residences by most building codes is 40 pounds per square foot. The average suspended ceiling will weigh from 10 to 12 pounds per square foot and a pad and carpet 3 to 4 pounds. The total superimposed load therefore will be:

\[
\begin{align*}
\text{Live load} & \quad 40 \\
\text{Suspended ceiling} & \quad 10 \\
\text{Carpet and pad} & \quad 3 \\
\hline
\text{Total} & \quad 53 \text{ pounds per sq. ft.}
\end{align*}
\]

The amount of reinforcing necessary may be determined from the design table, fig. 47. By reading down the column for a clear span of 15 feet to that section covering a joist spacing of 30 inches, we find a value of 60 pounds per square foot,
which is slightly higher than required in the present case. By following this line to the left, a reinforcing schedule of two 5/8" round bars is specified. Since the figure 60 is above the heavy line, we know that the shear value is less than 50 pounds per square inch and, hence, satisfactory. Had it been below the line, the shear would fall between 50 pounds per square inch and 75 pounds per square inch. In such a case hooked bars, as specified, should be used. Values are discontinued on the design tables when shear values reach 75 pounds per square inch.

The design is now completed. The joists over the recreation room will be 15' 8" long (allowing 4" bearing at each end), will have two 5/8" round by 15' 8" bars in each joist and will be spaced 30 inches center to center. The top slab will be 2 inches of 2,500 pound concrete.

Very often a partition will run parallel to the joists. In most cases it will be necessary only to use a double joist under this partition. However, for particularly heavy partitions or bearing walls, the weight of the wall should be added to the other superimposed loads and the joist designed as in the example above.

Figure 48 should be used in the design when drop filler tile are used.

SPECIFICATIONS—MATERIALS AND WORKMANSHIP

MATERIALS

Hollow tile units of burned clay used in constructing the tile beams shall conform to A. S. T. M. specifications C34-39 (2) for load-bearing structural clay tile and shall be of the design developed by the Clay Products Institute Research Fellowship at Iowa State College.

Hollow tile units of burned clay used in filling between the beams shall conform to A. S. T. M. specification for Structural Clay Floor Tile C57-39 and shall be scored on the two narrow and one broad face unless otherwise specified.

Mortar used for laying the beam tile and setting the floor tile shall consist of:

1 part portland cement
3 parts clean sand
1/4 part mortar mix or lime

Sand shall be well graded and free from such impurities as organic substances.

Coarse aggregate shall consist of pea gravel averaging 1/4"

in size and all passing a 1/2" screen and shall be clean and free from impurities such as organic substances.

Water shall be fresh, clean and free from alkali.

Concrete for the beams shall consist of 1 part of high-early strength portland cement, 3 parts of well-graded sand. The
**Fig. 47. Design table—flat span tile.**
### JOIST TILE DIMENSIONS

<table>
<thead>
<tr>
<th>JOIST SPACING</th>
<th>TYPING SPACER</th>
<th>WEIGHT OF FLOOR SYSTEM</th>
<th>REINFORCING STEEL</th>
<th>NUMBER AND SIZE OF BARS</th>
<th>RECOMMENDED AREA (SQU)</th>
<th>CLEAR SPAN IN FEET</th>
<th>SAFE SUPERIMPOSED LOAD (LBS PER SQ FT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>16&quot;</td>
<td>1/4</td>
<td>525</td>
<td></td>
<td></td>
<td>4.0 140 340 207 106 145 111 80 68 46 22</td>
<td>16</td>
<td>525 LBS PER SQ IN FOR HOOKED BARS</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5.0 345 294 251 193 151 116 93 72 52 32 22</td>
<td>17</td>
<td>525 LBS PER SQ IN FOR STRAIGHT BARS</td>
</tr>
<tr>
<td>22&quot;</td>
<td>1/4</td>
<td>587</td>
<td></td>
<td></td>
<td>6.0 373 319 269 207 155 115 96 76 56 34 22</td>
<td>17</td>
<td>525 LBS PER SQ IN FOR HOOKED BARS</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7.0 373 319 269 207 155 115 96 76 56 34 22</td>
<td>18</td>
<td>525 LBS PER SQ IN FOR STRAIGHT BARS</td>
</tr>
</tbody>
</table>

**Fig. 48.** Design table—drop filler tile.
Fig. 49. Joist hanger detail.

Fig. 50. Typical bearings on steel.
Concrete mix shall be kept as dry as possible and still work down around the steel bars.

Concrete for the topping shall consist of 6 1/2 gallons of water to the sack of cement and enough sand and gravel to make a workable, well-graded mix. Try a 1:2:3 mix (1 cement, 2 sand, 3 gravel).

Steel reinforcing bars used shall be round or square, deformed billet steel concrete reinforcing bars of intermediate grade conforming to A. S. T. M. specification A15-35.

**CONSTRUCTION**

**BEAM CONSTRUCTION**

The beams shall be constructed on a level plank. If it is necessary to camber the beams, the plank shall be blocked up to give the required camber at the center. The top half of the beam tile shall be well "buttered" with mortar and set against the preceding tile and tapped firmly against it leaving a mortar joint of not more than 3/8 inch.

An inch of concrete shall be slushed into the channels of the beam tile and the steel bars placed, being forced down to within half an inch of the bottom. The channel shall then be completely filled with concrete, care being taken to force it down around the bars.

The completed beams shall be straight and true with full and well-bonded head joints.
If under sides of beams are to be left exposed, all excess mortar shall be cleaned off after the floor has been erected and the joints pointed in a neat manner.

**FLOOR CONSTRUCTION**

The beams are to be spaced as indicated on the plan, so that the span tile has a bearing of at least an inch on the channel of the beam tile.

The span tile are to be well bedded in mortar as they are placed on the channels of the beams.

The concrete shall be poured as a continuous operation if possible. Care shall be taken to force the concrete down around the sides of the precast beams.

**CURING THE FLOOR**

When the floor is constructed during warm weather the beam and floor tile shall be well sprinkled before using.

After the beams are built they shall be kept sprinkled for 2 days.
If high-early strength cement is used in building the beams, they may be moved in 3 days. If regular portland cement is used, the beams should be allowed to set 7 days.

The span tile and beams shall be thoroughly soaked before the concrete topping is poured. The completed floor shall be kept sprinkled for 5 days.

During freezing weather the beams should be built in a shelter where a temperature of at least 50° F. can be maintained for not less than 48 hours.

The concrete topping, when poured, shall be at a temperature of at least 50° F. but not more than 100° F. The concrete shall be maintained at a temperature of at least 50° F. for not less than 72 hours after placing or until the concrete has thoroughly hardened.

**SHORING DURING CONSTRUCTION**

(Beams over 13 feet in length, but less than 18 feet)

After the beams are set on the wall, a row of shoring shall be placed, so as to set firmly against the center of each beam. This shall be left in place until after the floor tile are set and the concrete is poured and set.

(For beams over 18 feet in length)

Two rows of shoring shall be used. The shoring shall be placed at the third points and shall be left in place until after the concrete slab has set.

**COMPUTING COSTS**

**MATERIALS ESTIMATE**

**Beam tile:** Add 8 inches to clear span for length of beam.

**Span tile:** One more row than number of beams.

**Mortar**

Beam tile head joints: 7 cu. ft. per 1,000 lin. ft. of beam.

Span tile bed joints:

Drop filler type floor—None.

Flat span tile type floor—5 cu. ft. per 1,000 lin. ft. of beam.

**Concrete**

Beam channels: 80 cu. ft. per 1,000 lin. ft. of beam.

Topping

Drop filler type floor

1½" topping: 5¼ cu. yds. per 1,000 sq. ft. of floor plus

4 cu. yds. per 1,000 lin. ft. of beam.

2" topping: 7 cu. yds. per 1,000 sq. ft. of floor plus

4 cu. yds. per 1,000 lin. ft. of beam.

Flat span tile type floor:

2" topping: 7 cu. yds. per 1,000 sq. ft. of floor plus

8 cu. yds. per 1,000 lin. ft. of beam.
Temperature mesh: Area of floor in sq. ft. plus 6 percent for laps.

LABOR ESTIMATE

Building beams: 1 mason and 2 helpers can build 50 ft. per hr.
Setting beams: 4 laborers can place 130 ft. per hr.
1 mason and 1 helper can space and set 130 ft. per hr.

Setting span tile
Flat span tile: 1 mason and 1 helper can set about 50 tile per hr.
Drop filler tile: 1 laborer can set about 50 tile per hr.

Placing temperature mesh and top slab
Slab with smooth trowel finish: 4 cents per sq. ft.
Slab with rough finish: 2 cents per sq. ft.

SAMPLE COST ESTIMATE

Assume the following beam design is required:
Beams 30" o. c. 4"x12"x24" span tile. 2" topping. 2½" round bars per beam.
Assume the following labor scale:
Mason .............................................$1.375 per hr.
Mason’s helper ................... 0.50 “ “
Laborer ................................. 0.50 “ “
(This scale is approximately the present Iowa average)

I. Building beams

Materials
Beam tile—$83.80 per M ..................0.0838 per ft.
Mortar—0.007 cu. ft. @ 0.37 per cu. ft. ..0.0026 “ “
Concrete—0.08 cu. ft. @ 0.296 per cu. ft...0.0237 “ “
Steel—two ½” round bars=1.33 lbs. per ft.
@ 0.0325 per lb. .....................0.0430 “ “

Labor
1 mason and 2 helpers @ $2.375 per hr.
can set 50 ft. per hr. ......................0.0475 “ “

Total cost of beams per ft. .............0.2006 per ft.

II. Placing beams

4 laborers @ $2.00 per hr. can place 130 ft. per hr. ..................0.0154 per ft.
1 mason and 1 helper @ $1.875 can space and set 130 ft. per hr. .............0.0144 “ “

Total cost of placing beams .............0.0298 “ “
Total cost of beams in place ...........0.230 per ft.
III. Placing span tile

Materials
- Tile 4"x12"x24" @ $176.00 per M ............ 0.1760 per ft.
- Mortar—0.05 cu. ft. per ft. @ 0.37 ............ 0.0185 " "

Labor
- 1 mason and 1 helper @ $1.875 per hr.
  can set 50 tile per hr. ......................... 0.0375 " "

Total cost of span tile in place ............ 0.2320 per ft.

IV. Placing temperature mesh and top slab

Material
- Mesh—6"x6" No. 10 x No. 10 welded wire
  mesh 2.65 sq. ft. @ 0.0135 .................. 0.036 per ft.
- Concrete—0.255 cu. yds. @ $8.00 per
  cu. yd. .................................................. 0.204 " "

Labor
- Placing mesh ........................................ 0.050 " "
- Placing slab (trowel fin.) ..................... 0.100 " "

Total cost of mesh and slab in place ... 0.390 " "

V. Cost of floor in place

Beams ........................................ 0.230 per ft.
Span tile ........................................ 0.232 " "
Top slab .......................................... 0.390 " "

Total cost of floor per lin. ft. of
  beam ........................................ 0.852 per ft.
Cost per sq. ft. of floor 0.852/2.5=$0.34.

The above cost estimate is based on average material prices
and average union wage scales. The number of hours of
labor were averaged from cost studies made on actual jobs
where an accurate and detailed check was kept on the cost
of construction. The costs cited do not include charges for
overhead nor profit for the contractor.

CONSTRUCTION DETAILS AND METHODS

Experience has shown that the precast tile beam floor can
be installed easily and rapidly. The following description of
the procedure illustrated with pictures of actual jobs should
give the average small builder sufficient information to suc-
cessfully install the floor in all jobs except large buildings
where unusual loading conditions require special framing
details.

BLOCK DESIGN

The basic unit in the floor system is the T shaped beam
tile. The various units now available are listed below.
1. The standard beam tile is 8" wide, 6" deep and is available in 3", 6", 9" and 12" lengths, permitting the builder to get the desired length of beam.

2. The heavy-duty beam tile is 10" wide, 8" deep and is also available in 3", 6", 9" and 12" sections.

3. The standard span tile is 4" deep, 12" wide, and 12", 16", 20" or 24" in length.

4. Drop filler tile. This unit, used when a flat ceiling is desired, is 12" wide at the top and 10" wide at the bottom. It is 6" deep and 12" long.

BUILDING BEAMS

The builder should make some preparation for equipment for building the beams. On a large job considerable time should be spent in organizing the procedure in building the beams. On the ordinary small job the builder selects several 2"x8" or 2"x10" planks that are fairly straight and have one good straight side, tacks a 1"x4" board on the straight side allowing it to project up to form a guide in building the beams. On a job of any size it is wise to have enough plank to take care of 2 days’ building. If quick-setting cement is used, the beams built the first day can be slipped off the plank and the form used for the third day’s building. Some builders lay the plank out on the ground while others prefer to set them up on two or three tile. Still other builders prefer to rack the beams and build one above the other. Each builder with a little experimenting will find the method he likes best. If the beams are built in a central yard and trucked to jobs, permanent metal forms would be a good investment.

The procedure followed in building beams is given below. If the specifications are adhered to, strong rigid beams will be obtained.

Fig. 53. Raised platform for building beams.
Thoroughly wet the beam tile before using. This can be done either by dipping the tile in a barrel of water or thoroughly sprinkling with a hose. This should be done at least $\frac{1}{2}$ hour before using so no water will be present on the surface of the tile.

The top half of the beam tile should be well “buttered” with mortar, set against the preceding tile and tapped firmly against it leaving a mortar joint of not more than $\frac{3}{8}$ inch. To obtain rigid beams, the top joint on the beam tile should be well filled. After the tile are tapped together the mortar should not be cut off the top but left projecting. Where a finished appearing ceiling is desired, the bottom joint of the beam tile should not be “buttered” with mortar but left open.
and tuck pointed later. If the ceiling will be covered or a finished appearance is not essential the entire end of the beam tile can be “buttered.”

An inch of concrete should be slushed into the channels of the beam tile and steel bars placed, being forced down to within ½ inch of the bottom. The channels shall then be completely filled with concrete, care being taken to force it down around the bars.
The top of the concrete in the channel should be leveled off so that it does not project above the tile channel. A rough surface on the top of the concrete in the channel is desirable. The beams should be kept sprinkled and under cover for 2 days and may be moved the third. It is recommended that they be allowed to stand 5 days if possible. Unless quick-hardening portland cement is used in building the beams, they should be kept wet for at least 5 days, and they may be moved in 7 days.
HANDLING AND PLACING BEAMS

The method of handling and placing the beams will vary with the size and type of job. Four men can carry the beams from the stack to the job with ease. On small jobs the simplest method is to slide the beams across the span on a 2"x10" plank. The foreman should have the beam spacing...
marked on the wall so that the beams can be set in place without rehandling.

The second method for setting the beams and one that speeds up a large job is to use a two-wheel dolly with each wheel resting on a 2''x10'' plank. One end of the beam is set on the dolly so as to extend about 2 feet beyond the platform. The men can then shove on the other end of the beam. When the end of the beam has reached the other bearing wall a rope or chain is thrown around the end of the beam in front of the dolly but back at least 8 inches from the end of the beam. The men on the rope or the chain lift enough to permit the dolly to be removed and the beam is then lowered to place. A mortar bed should be placed on the wall between the marks made by the foreman. There are
several methods of hoisting beams to a second floor, third floor or roof. For small jobs the simplest method is to build the beams on the first floor and lift them to the second floor, using the mason’s scaffolding. Two men will lift one end of the beam 4 feet to the scaffolding on one side of the room, two men will then lift the other end of the beam 4 feet up
to the scaffolding on the other side. The process is then repeated in lifting the beam into position on the wall. The scaffolding may be replaced by a small movable platform. On jobs of any size the beams can be hoisted by block and tackle, placing a support around each end of the beam. Another method is to spread two planks against the building and pull the beams up on this ramp.

SHORING UNDER LONG SPANS

Beams with spans over 13 feet should be supported by a row of shoring during the time the span tile and concrete are
being placed. This will insure the builder of a good straight ceiling. For beams over 18 feet in length, two rows of shoring at the third points are recommended.

PLACING SPAN AND FILLER TILE

The span tile should be well bedded in mortar and placed in position with ends resting on the channels of the beams. The mortar joint should also be carried between the span tile where the span is over 16 feet or the live load over 50 pounds per square foot. The drop filler tile need not be set on a bed of mortar, however.

PLACING TOPPING

The concrete topping should be poured as a continuous operation if possible. Care should be taken to force the concrete down around the sides of the precast beams. When the floor is constructed during warm weather the beams and span tile should be well sprinkled before the concrete is placed. The completed floor should be kept sprinkled for 5 days. The concrete topping when poured should be at a temperature of at least 50° F. but not more than 100° F. The concrete should be maintained at a temperature of at least 50° F. for not less
than 72 hours after placing or until the concrete has thoroughly hardened.

TEMPERATURE STEEL

Reinforced mesh or bars should be placed in the top of the slab on all exposed work.

CONTINUOUS BEAMS

Beams continuous of several supports should be reinforced to resist negative bending moment. A bar should be placed not closer than 1 inch from the top of the slab. This reinforcement should be placed on all jobs where there is a sup-

Fig. 70. Heat risers are placed before span tile are laid.

Fig. 71. Concrete slab in place with chases for heat ducts.
port even if it is just a residence with two spans. Placing of this steel will prevent cracking above the support.

**LOCATING DUCTS AND PLUMBING MEMBERS**

With any type of permanent construction, careful planning of the utilities is necessary. The heating contractor must make an accurate layout and indicate where the openings are to be in the floor. This is also true of the plumbing. The heating engineer should make his layout with a framing plan for the tile joist before him so that he can make his

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**Fig. 72.** Placing conduit after span tile are in place.

**Fig. 73.** Showing use of hangers to frame around stair wells.
risers come up between the beams, or if it seems advisable the framing plan should be made from the heating plan, locating the necessary openings for risers.

STAIR WELLS AND OTHER OPENINGS

Framing around stair wells and other openings has been simplified by the development of a bent bar hanger shown in fig. 73. The bent hanger bars are placed over a beam tile and in the channel of the beam being cast to get the correct location in the channel. The beam is then completed in the usual manner. For openings in residential work the steel in the header beam carrying the load should be increased one size, that is, from $\frac{5}{8}$ to $\frac{3}{4}$ or from $\frac{1}{2}$ to $\frac{3}{4}$ inches. On heavy loads or where the beams hanging on the supporting beam are over 6 feet in length the design should be checked. In some cases it may be necessary to double the supporting beams. This would mean increasing the length of the hanger bars. In most cases where more than one joist rests on the header joist it is wise to check the design thoroughly.

INSULATION AND SUSPENDED CEILINGS

The tile floor construction has an insulation factor of .365. In most cases where the floor is used for a roof additional insulation will be desired. This can be obtained by placing insulating board on the roof slab before the built-up roof is placed or by using concrete with light-weight aggregate on the top slab and then placing the built-up roofing, or by placing insulation below the beams. If the drop filler type of tile is used, the insulating board can be cemented thoroughly
to the underside of the floors. If the long-span tile are used and a suspended ceiling installed, fill insulation can be placed between the suspended ceiling and the slab. A suspended ceiling can be installed very easily where this is desired. Wires are looped and dropped between the span tile. Light weight steel channels can then be attached to the wires and rib lath to the channels. The manufacturer’s specifications for weight and spacing of channels and weight and spacing of rib lath should be adhered to.

DECORATIVE TREATMENT

Any popular floor finish can be used with this construction. If wood floors are desired, either sleepers or clips to hold sleepers are placed in the topping. Carpet, linoleum or rubber tile can be laid on the concrete surface. Tile, terrazzo or colored concrete can be placed as the topping is being finished.

Ceilings, or the underside of the floor, can be left the color of the tile or may be decorated. The proportions of the beams are well adapted to producing an attractive, beamed ceiling effect. Figure 29 shows a ceiling with the beams spaced 28 inches o. c. The mortar may be colored to match the tile to obtain a continuous beam effect. The ceiling may be painted or stenciled to harmonize with various interior treatments.

CONSTRUCTION AT EXTERIOR WALLS

It is recommended that at least \( \frac{1}{2} \) inch of air space be left between the ends of the joists or slab and the back of the exterior facing material. Any expansion of the floor slab will then be taken up in this space, thus eliminating the possibility of the exterior face of the wall being pushed outward as a result of such expansion.
BIBLIOGRAPHY


